A NEW SEMI-ANALYTICAL LAG-TIME EQUATION FOR APPLICATIONS IN THE KANSAS CITY METROPOLITAN AREA

By

Ricardo A. Gamarra Zapata

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Chairperson Bruce M. McEnroe

C. Bryan Young

Alfred D. Parr

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The Thesis Committee for Ricardo A. Gamarra Zapata

certifies that this is the approved version of the following thesis:

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in the Kansas City Metropolitan Area

Chairperson Bruce M. McEnroe

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Abstract

Hydrologic methods that relate peak discharge to storm rainfall require as inputs time parameters that describe how fast the watershed responds to rainfall events. Two of these parameters are lag time (T_L) and time of concentration (T_C). In this report we present new equations for the estimation of T_L and T_C that are applicable to both urban and rural watersheds. Physical reasoning was used to formulate a semi-analytical equation that accounts for the major relevant factors. The new equation for lag time was derived from the Manning equation for hydraulic friction, the rational equation for peak flow and a rainfall intensity-duration relationship. The relevant factors include length and slope of the main channel, the average width of the watershed, and two measures of urbanization: the fraction of impervious surface area and the fraction of the main-channel length that is enclosed or paved. Lag time depends mainly on the three channel characteristics. The two watershed characteristics are significant but less influential. The equation was calibrated with data from the analysis of rainfall and stage data for 30 gage sites from the ALERT flash-flood system in the Kansas City metropolitan area and the analysis of the physical characteristics of the watersheds. An approximate analysis lends some support to the approximated NRCS-recommended relationship $T_C = 5/3\ T_L$. This relationship was used to obtain the T_C equation. Other equations developed by regression methods proved to be less satisfactory. Because the semi-analytical equations have a solid physical basis, they should give reasonable results for smaller watersheds and for more densely developed watersheds.

Acknowledgements

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To my parents, Ramiro and Virginia, and to my brother, Edu, thank you for your unconditional love and support.

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Chapter 1

Introduction

1.1. Time parameters and local practices

Hydrologic methods that relate peak discharge to storm rainfall require time parameters that describe how fast the watershed responds to rainfall events. Two such parameters are lag time (T_L) and time of concentration (T_C), which are affected by geomorphic and climatologic characteristics of the watershed. Lag time is needed to simulate flood hydrographs and time of concentration is needed to estimate peak flows by the rational method. Several different definitions for lag time and time of concentration can be found in the literature. In this study, T_L and T_C are defined as in the Natural Resources Conservation Service's (NRCS) National Engineering Handbook (NRCS, 2010). Time of concentration is defined as the time it takes a particle of water at the hydraulically furthest point in the watershed to travel to the outlet. Lag time is defined, in a hydrograph analysis context, as the time difference between the center of mass of excess rainfall and the peak discharge at the watershed outlet. Greater values of these parameters indicate that, for a given rainstorm, the watershed will respond slower and consequently the peak discharge will be smaller. Urbanization decreases the response time by reducing the frictional resistance for flow.

Lag time can be determined directly from rainfall and stage records with short time intervals between data values. However, time of concentration cannot be determined from gaging data, and there are no practical methods to measure it in the field. Time of concentration can be estimated with hydraulic calculations, but these calculations generally involve numerous approximations and uncertain inputs. Alternatively, time of concentration can be estimated from lag time. According to NRCS (2010), time of concentration equals five-thirds of lag time.

Most cities in the Kansas City metropolitan area require storm drainage infrastructure to be designed in accordance with the Section 5600 design guidance document of the Kansas City Metro Chapter of the American Public Works Association (KC-APWA). Following Section 5600, time of concentration is estimated by a velocity method which segments the flow path for different stages of flow and channel conditions. T_C is computed as the sum of overland flow time to the most upstream point of entry into the system (inlet time) and the flow time in the system to the outlet (travel time). Lag time is approximated as three-fifths of the time of concentration. Inlet time is calculated by the FAA equation for sheet flow (FAA, 1970) while travel time is calculated using the Manning equation for uniform flow. Previous studies in Johnson County (McEnroe and Young, 2011) have shown that using the FAA equation overestimates the inlet time because a significant part of the flow to the most upstream inlet is shallow concentrated flow rather than sheet flow. Hydraulic estimates of travel times in natural channels can be unreliable due to channel irregularities and uncertainties in Manning roughness coefficients.

The Kansas Department of Transportation’s Road Design Manual (KDOT 2011) provides regression equations for lag time and time of concentration for urban and rural areas. These equations consider the length and slope of the main channel and, for urban watersheds, the percentage of impervious area. The lag-time equation for rural watersheds was developed from data for 19 USGS gages in rural Kansas with the watershed areas from 1 mi^2 to 14 mi^2 (McEnroe and Zhao, 2000). The urban lag-time equation was developed from data for 14 gages from the Johnson County ALERT system with watershed areas from 0.3 mi^2 to 28 mi^2 (McEnroe and Zhao, 2001). Since the completion of the 2001 study, more gaging stations have been added on smaller streams and another 14 years of data have been collected. KDOT's equations for time of concentration yield T_C values equal to $5/3 \cdot T_L$, in accordance with NRCS guidance.

1.2. Literature review

A recent paper by Gericke and Smithers (2014) provides a comprehensive review of methods for estimating lag time, time of concentration and other time parameters. This review also lists the different definitions for T_L and T_C found in the literature and tries to clear up some of the resulting confusion.

Methods for estimating T_L and T_C can be classified as hydraulic or empirical. Hydraulic methods are derived from either uniform-flow theory or wave mechanics. This category includes the velocity method, which assumes steady-state uniform flow, usually at bankfull capacity. Also included are equations derived from kinematic wave analysis, which are applicable mainly to overland flow. Most kinematic-wave equations idealize the watershed as a planar surface with non-converging flow (Welle and Woodward, 1986). In most kinematic-wave equations, the term referred to as time of concentration is actually the time to equilibrium rather than the travel time at steady state. The time to equilibrium is the time needed for rainfall of constant intensity to produce a constant maximum discharge at the watershed outlet, starting from a dry surface (Eagleson, 1970).

Empirical methods relate lag time or time of concentration to watershed characteristics by fitting a model, usually a power function, to data by regression analysis. Empirical equations generally provide estimates of the total lag time or time of concentration, accounting for both overland flow and channel flow (Gericke and Smithers, 2014). Many researchers use physical reasoning to select the variables and structure the equations. Fitted equations with forms based in part on physical reasoning are termed semi-analytical equations. A common grouping of variables included in many equations is $L/S^{0.5}$ because the time required for a flood wave to travel through a channel reach is proportional to this quantity.

Empirical formulas fitted to data from a particular geographic region might not work as well outside that region due different geomorphological and climatological characteristics. Gericke and Smithers (2014) present 19 equations for T_C and 21 equations for T_L, many of which do not account for the effects of urban development and/or use different definitions for T_C and T_L.

1.3. Overview of the study

A semi-analytical approach was used to formulate a general equation for urban lag time that is dependent on five relevant watershed characteristics. The new lag-time equation was calibrated using of rainfall, stage and watershed characteristics data for 30 gage sites from the ALERT flash-flood warning system. The average lag time for each gaged watershed was determined by analyzing rainfall and stage data for large rainfall events. The new equation for lag time was fitted to the data for the gaged watersheds. Equations of other forms, which proved to be less satisfactory, were also developed from the data using regression analysis.

Chapter 2 discusses the procedures used to select the gaged sites, measure the physical watershed and channel characteristics and determine the lag times. In Chapter 3 a general semi-analytical equation for urban lag time is derived and fitted to the Kansas City data. Chapter 4 describes the development of other lag-time equations by regression analysis. Chapter 5 explains why the new semi-analytical lag-time equation is recommended and compares it to the equation for urban lag time derived from ALERT-system data in 2001. The conclusions are presented in Chapter 6.

Chapter 2

Physical Characteristics and Lag Times of Gaged Watersheds

2.1. Selection of gaged watersheds

The 30 gage sites selected for this study are located in the Kansas City metropolitan area and belong to the ALERT flood warning system managed by the City of Overland Park, Kansas. Fifteen of the gages are in Johnson County, Kansas; nine are in Clay County, Missouri; four are in Jackson County, Missouri; and two gages are in Platte County, Missouri. All of the selected sites have consistent and reliable rainfall depth and water-level records. There are no significant impoundments upstream of these sites. On streams with multiple water-level gages, the gage site furthest upstream was selected, with one exception. The watersheds have a wide range in size (113 acres to 11.11 mi^2) and amount of development, measured as the ratio of impervious surface area to total drainage area (1.2% to 49.6%). Table 2-2 lists the selected gage sites and Figure 2-1 shows the watershed boundaries on a map.

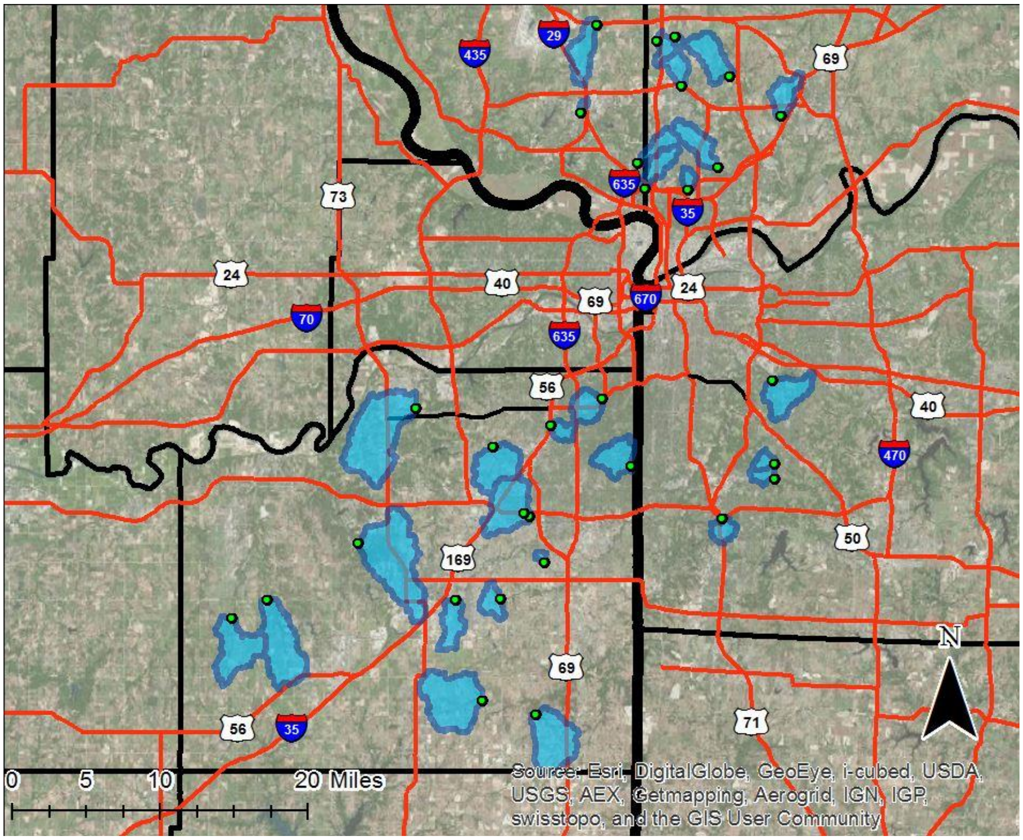

Figure 2-1. Locations of selected gage sites and watersheds

2.2. Geospatial data and analysis

Geospatial data for this study were compiled from a number of sources. Johnson County's Automated Information Mapping System (AIMS) office provided data in ArcGIS format for the watersheds located in Kansas. These data consisted of aerial imagery, road centerlines, general land use, 2-ft elevation contours, pavement edges, water bodies, stormwater drainage lines, building polygons, driveway centerlines and a digital elevation model (DEM) with a cell size of 3 feet. All of the data were provided in the StatePlane coordinate system for Kansas North FIPS 1501 (US Feet).

For the Missouri watersheds, the City of Kansas City, Missouri (KCMO) provided GIS data that included road centerlines, stormwater drainage lines, inlets, impervious surfaces polygons, 2-ft elevation contours, water bodies, outlets and manhole and outfall point locations. KCMO also provided DEMs with one-meter cell size for Clay and Platte counties, and LiDAR points of bare-earth terrain for Jackson County, which were later converted to raster data with the same resolution as the other DEMs. In addition, the cities of Raytown and Gladstone provided stormwater drainage maps. The coordinate system for all Missouri data was StatePlane Missouri West FIPS 2403 (US Feet). Information on impervious surfaces were not provided for areas in Missouri outside the City of Kansas City. In these areas, we estimated imperviousness from a 30-m National Land Cover Database (NLCD) 2011 Percent Developed Imperviousness layer.

Geospatial data were processed in ESRI® ArcMap version 10.1 and the corresponding version of Arc Hydro Data Model and Tools. First, any vector data containing information on the flow paths for streams (whether natural or from the stormwater drainage network) upstream of the gages were used to recondition the DEMs and "burn" channels onto them. This was done mostly to force flow through enclosed conduits, instead of having runoff follow the downward gradient of the ground surface. The watersheds were delineated with the following tools from Arc Hydro: “Fill sinks” to eliminate depression in the DEMs, “Flow Direction”, “Flow Accumulation”, “Stream Definition” with the default inputs (threshold depending on size of DEM), “Stream Segmentation”, “Catchment Grid Delineation”, “Catchment Polygon Processing”, “Drainage Line Processing”, “Adjoint Catchment Processing” and “Point Delineation.” Delineation of the longest channel in the watershed and calculation of its slope were done using the “Longest Flow Path” and “Flow Path Parameters from 2D Line” tools. Other watershed characteristics were obtained with basic ArcToolbox functions.

2.3. Watershed characteristics

The physical characteristics chosen to describe the selected watersheds are listed in Table 2-1. In addition to drainage area, these characteristics include three measures of length, two measures of slope, an average width, a measure of imperviousness, and a measure of channel modification. The values of these characteristics for each site are listed in Table 2-2.

Table 2-1. List of watershed characteristics

Characteristic	Symbol	Unit	Description
Area	A	Acres	Watershed drainage area upstream of gage.
Length of longest flow path	L	Feet	Length of the longest flow path from a point on the drainage divide to the watershed outlet.
Length to centroid	L_c	Feet	Length along the longest flow path from the watershed outlet to the point closest to the watershed centroid
Average flow path length	L_{avg}	Feet	Average length of the flow path from a point in the watershed to the watershed outlet
Average width	W	Feet	Average width of the watershed, defined as A/L.
Average slope	S	ft/ft	Average slope of the longest flow path. Calculated as the elevation difference between the drainage divide and the watershed outlet, divided by L.
Average slope (10% - 85%)	$S_{10\text{-}85}$	ft/ft	Average slope of the longest flow path between the 10% and 85% points of the flow path length.
Paved channel ratio	R_c	Dimensionless	Fraction of the longest flow path that is enclosed or has a paved bottom with low frictional resistance. Channels with side gabions or concrete lined channels with rocky or soil bottoms are not considered “paved”.
Impervious area ratio	R_i	Dimensionless	Fraction of watershed area covered by impervious surfaces (pavements and buildings).

Table 2-2. Selected gage sites and watershed characteristics

ID	Name	State	Area (ac)	L (ft)	L_c (ft)	L_{avg} (ft)	W (ft)	S (ft/ft)	S_{10-85} (ft/ft)	R_c	R_i
1140	143rd @ Indian Creek	KS	1554	17663	7398	9057	3833	0.0053	0.0054	0.356	0.339
1400	Waterford (N. Br. Indian Cr.)	KS	3415	21542	8981	12700	6906	0.0081	0.0076	0.429	0.427
1450	I-435 @ Quivira	KS	678	11702	5572	6235	2524	0.0135	0.0151	0.621	0.480
1650	Pflumm @ Tomahawk Creek	KS	1010	10188	3317	5456	4318	0.0094	0.0086	0.313	0.255
1680	Wilshire Woods	KS	170	4697	2430	2688	1572	0.0178	0.0191	0.652	0.326
2090	191st St. @ E. Wolf Cr.	KS	3864	26707	12528	14850	6303	0.0062	0.0052	0.004	0.039
2220	Lackman @ Wolf Cr.	KS	5004	32793	15686	18626	6646	0.0039	0.0029	0.007	0.020
2540	96th & Brighton East Fork Shoal Creek	MO	2156	22376	11452	12209	4197	0.0090	0.0064	0.027	0.098
2600	NE 112th Ter @ Rocky Branch Creek	MO	182	5367	2177	2797	1475	0.0149	0.0157	0.581	0.234
2640	NE Vivion Rd @ Rock Creek	MO	412	7342	3188	4069	2446	0.0193	0.0188	0.560	0.307
2700	Hickman Mills Dr & I-470	MO	801	8823	2541	5234	3957	0.0160	0.0170	0.193	0.339
2720	Elm Rd @ White Oak Creek Trib	MO	530	9857	3829	4970	2344	0.0134	0.0142	0.759	0.241
2730	E 83rd St @ White Oak Creek	MO	511	8919	3456	4772	2496	0.0153	0.0147	0.640	0.238
3020	69th @ Quail Crk Trib to Turkey	KS	660	10523	5177	5679	2732	0.0135	0.0135	0.621	0.311
3160	79th St @ Little Mill Creek	KS	2806	16796	6097	10040	7276	0.0082	0.0106	0.282	0.372
3170	Woodland @ Clear Creek	KS	7108	57155	31528	30066	5417	0.0049	0.0039	0.021	0.138
3250	119th @ Little Cedar Creek	KS	7093	42251	15896	19830	7313	0.0055	0.0052	0.087	0.195
3310	143rd @ Kill Creek	KS	4455	40910	22622	22949	4744	0.0040	0.0038	0.068	0.062
3350	151st @ Spoon Creek	KS	3384	33349	15181	18650	4420	0.0045	0.0037	0.002	0.012
3660	Skyview @ 2nd Creek Trib	MO	1891	29011	15481	15997	2839	0.0055	0.0039	0.101	0.183
3690	Summit @ First Creek	MO	711	10441	3374	5757	2966	0.0066	0.0072	0.107	0.209
3720	Hwy 152 @ Upper Shoal Creek	MO	946	13498	7464	7816	3052	0.0097	0.0128	0.169	0.308
3840	NW Waukomis @ Old Maids Creek	MO	1038	21514	13629	13202	2101	0.0114	0.0091	0.141	0.296
3900	NW Vivion @ East Creek	MO	1221	21892	11271	11647	2429	0.0118	0.0123	0.196	0.285
3940	N Jackson Dr @ Rock Creek	MO	1769	23384	11956	12927	3296	0.0083	0.0060	0.144	0.318
3980	NE 79th @ East Fork Little Shoal Creek	MO	1389	21466	10162	10640	2819	0.0111	0.0099	0.219	0.290
4080	Blue Ridge Cutoff @ Round Grove Creek	MO	2358	18736	6760	9581	5482	0.0127	0.0130	0.289	0.256
4150	NW 80th @ Walnut Creek	MO	113	5445	2062	2526	908	0.0149	0.0131	0.417	0.496
5050	Lee Blvd @ Dykes Branch	KS	1951	17110	7788	8431	4968	0.0117	0.0099	0.530	0.337
5700	Martway @ Rock Creek	KS	1419	14172	6615	7735	4362	0.0123	0.0119	0.736	0.414

2.4. Relation between time of concentration and lag time

According to the NRCS National Engineering Handbook (2010), in an average natural watershed with an approximately uniform distribution of runoff, lag time and time of concentration are related by

$$\mathrm{T_L} = 0.6\ \mathrm{T_C} \tag{2-1}$$

While little solid evidence can be found to support the relationship in (2-1) and many authors have proposed different values for the coefficient, the NRCS relationship is widely accepted in engineering practice.

The NRCS National Engineering Handbook states that lag time can be thought of as an area-weighted average travel time from any point within the watershed to the watershed outlet. A cell in a DEM could be considered such a point. It follows that the average travel time from all the cells on a DEM within a watershed could be considered the lag time for that watershed. Assuming the travel time from any cell to the outlet to be directly proportional to the length of the flow path from that cell to the outlet (i.e. considering the average flow velocity on each flow path to be the same), we can make the following statement:

$$\frac{\mathrm{T_L}}{\mathrm{T_C}} = \frac{\overline{\mathrm{T}}_\mathrm{t}}{\mathrm{T_C}} \approx \frac{\mathrm{L_{avg}}/\overline{\mathrm{V}}}{\mathrm{L}/\overline{\mathrm{V}}} = \frac{\mathrm{L_{avg}}}{\mathrm{L}} \tag{2-2}$$

For the dataset of this study, the average ratio of $\mathrm{L_{avg}}/\mathrm{L}$ is 0.54, as shown in Figure 2-2. This ratio provides a rough estimate of the ratio $\mathrm{T_L}/\mathrm{T_C}$, based on the coarse approximation of a constant flow velocity throughout the watershed. This result provides some support for the

NRCS-recommended approximation $T_L/T_C = 0.6$. However, we note that the value of T_L/T_C for a given watershed depends on the watershed's shape, the spatial distribution of urban development and other factors. The relationship between these two time parameters should be studied further.

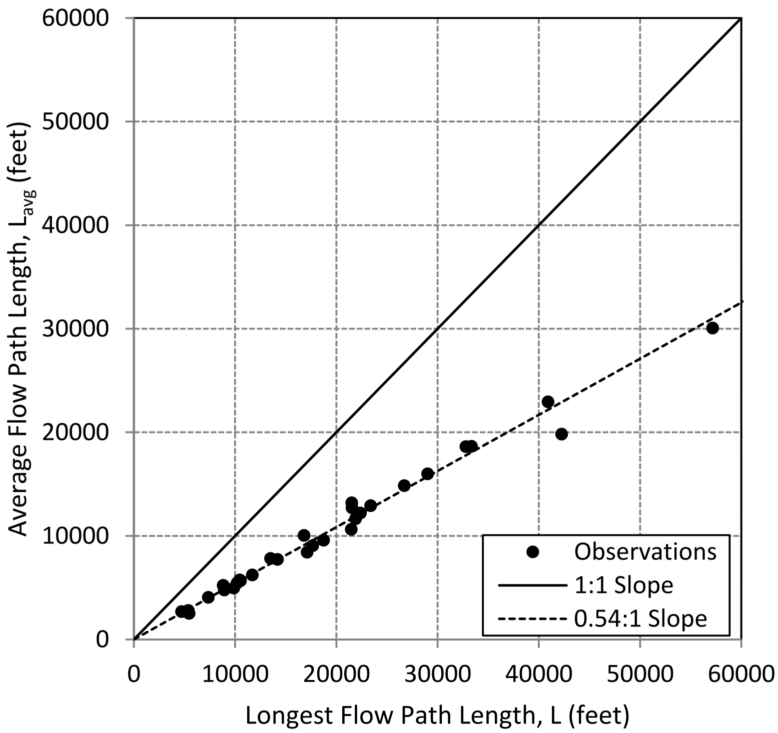

Figure 2-2. Relationship between L_{avg} and L

2.5. Selection of rainfall-runoff events

The City of Overland Park, Kansas, provided rainfall and water-level records for the selected gages. The gages located in Kansas have records starting as early as 1986 while most of the Missouri gages have records starting around 2010. Our study considered data through December

2014. Rainfall depths were measured by tipping-bucket gages that record the exact time for each 1-mm increment of rainfall. Water levels were recorded at 0.05-ft intervals in most cases.

Rainfall-runoff events were discarded if the rise in stage could not be explained by the recorded rainfall. Long-duration and low-intensity events were dropped because reliable lag-time estimates could not be obtained for these events. Short, intense events with only one clear peak stage were preferred. Events were considered separate if the water level returned to a stage close to its initial depth before more rainfall was recorded. A total of 220 individual events were selected for analysis at the 30 gages sites.

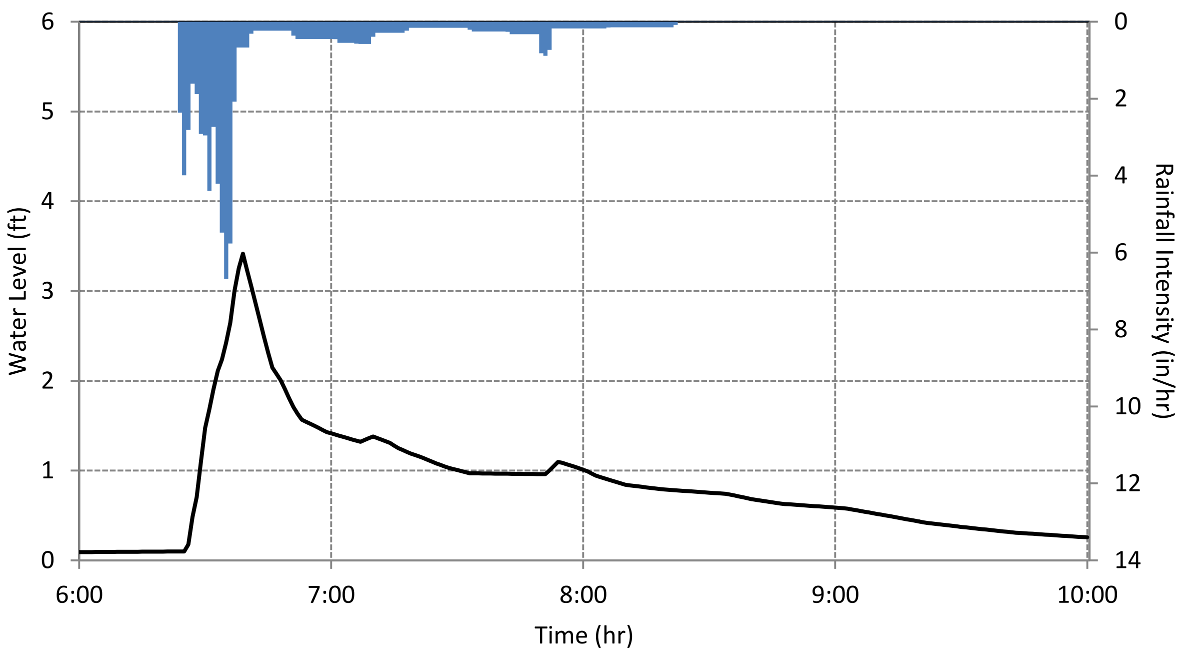

Figure 2-3. Sample hyetograph and hydrograph for event on 8/26/2005 at site 1680

2.6. Determination of lag times for selected events

Lag times for individual events were determined by simulation using the U.S. Army Corps of Engineers' Hydrologic Modeling System (HEC-HMS ver. 3.5). The simulations were done with the purpose of finding a lag time for each event that would replicate the observed watershed response most closely. More specifically, the goal was to find the lag time that would cause the peak discharge to occur at the same time as the observed peak water level.

HEC-HMS simulations work by applying a storm model to a basin model for a specified simulation period. The storm models contained recorded cumulative precipitation depths interpolated to 1-minute intervals. Rainfall was distributed uniformly over the entire watershed. The watershed was modeled as one basin with impervious areas uniformly distributed. Incremental runoff depths were computed by the NRCS curve-number method. In accordance with KCAPWA Section 5600, the curve number for the pervious areas was set to 74, which is representative of grass-covered ground in good condition with soils in hydrologic group C. The initial abstraction was set to one-fifth of the maximum potential retention, the NRCS-recommended default setting. HEC-HMS assumes that all rainfall on impervious surfaces becomes direct runoff with no losses. Direct runoff was transformed into discharge at the watershed outlet by the NRCS unit hydrograph method. Baseflow was set to zero in all cases. The computational time interval was set to one minute. A simulation run time was chosen that allowed the simulated discharge to return to zero or close to zero.

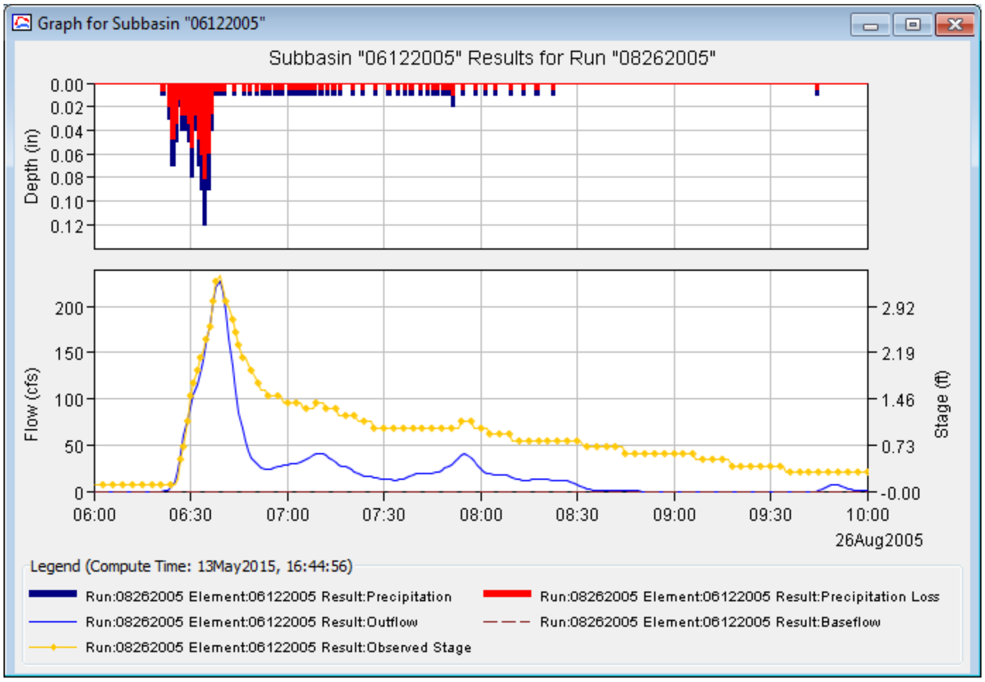

Figure 2-4. Sample simulation run for event on 8/26/2005 at site 1680

The lag time for each event was determined by calibration. A calibration was considered successful if the chosen lag time produced a simulated discharge with a peak that occurred at the same time as the observed peak stage. In addition, the overall shape of the simulated hydrograph was checked for similarity to that of the recorded water-level record.

2.7. Average lag times for watersheds

After calibrating the lag times, a few events were dropped from consideration the calibrated lag times were inconsistent with the results for the other events from the same site. Gage sites with few events and no consistency in the lag times were also discarded. The calibrated lag times for different storms showed considerable variation at some gage sites. The differences in lag times are attributable to differences in rainfall spatial patterns and other factors. Event lag times showed greater consistency on the smaller watersheds probably because the gage rainfall was more representative of the average rainfall over the watershed. The median lag time from the set of events for each watershed was calculated and considered to be the lag time for the site. The median was chosen as the representative lag time because it is a measure of central tendency that is not greatly influenced by outliers. The median lag time for each site is shown in Table 2-3.

Table 2-3. Median lag times for selected watersheds

Site ID	Name	Lag time (min)
1140	143rd @ Indian Creek	32.5
1400	Waterford (N. Br. Indian Cr.)	41
1450	I-435 @ Quivira	18
1650	Pflumm @ Tomahawk Creek	26
1680	Wilshire Woods	6
2090	191st St. @ E. Wolf Cr.	73
2220	Lackman @ Wolf Cr.	152
2540	96th & Brighton East Fork Shoal Creek	57
2600	NE 112th Ter @ Rocky Branch Creek	15
2640	NE Vivion Rd @ Rock Creek	11
2700	Hickman Mills Dr & I-470	18
2720	Elm Rd @ White Oak Creek Trib	7
2730	E 83rd St @ White Oak Creek	10
3020	69th @ Quail Crk Trib to Turkey	13
3160	79th St @ Little Mill Creek	38
3170	Woodland @ Clear Creek	98
3250	119th @ Little Cedar Creek	72
3310	143rd @ Kill Creek	118.5
3350	151st @ Spoon Creek	139
3660	Skyview @ 2nd Creek Trib	103
3690	Summit @ First Creek	39
3720	Hwy 152 @ Upper Shoal Creek	43
3840	NW Waukomis @ Old Maids Creek	55.5
3900	NW Vivion @ East Creek	37
3940	N Jackson Dr @ Rock Creek	33
3980	NE 79th @ East Fork Little Shoal Creek	56.5
4080	Blue Ridge Cutoff @ Round Grove Creek	31
4150	NW 80th @ Walnut Creek	17
5050	Lee Blvd @ Dykes Branch	28
5700	Martway @ Rock Creek	12

Chapter 3

A General Semi-Analytical Relationship for Lag Time of an Urban Watershed

In this chapter, a semi-analytical relationship for the lag time of an urban watershed that accounts for several of the most relevant characteristics is presented. This general relationship is calibrated with the data for the 30 watersheds listed in Table 2-3.

3.1. Derivation

The approximation that lag time is a fixed fraction of the time of concentration, defined as the total time of flow along the longest flow path, is widely accepted in engineering practice (e.g., NRCS 2010). It follows that lag time can be considered directly proportional to the length of the longest flow path and inversely proportional to a representative velocity on this flow path:

$$T_L \propto \frac{L}{V} \tag{3-1}$$

where

T_L = lag time

L = length of the longest flow path

V = a representative velocity on the longest flow path

The Manning friction equation can be used to relate the representative velocity to representative values of hydraulic radius, slope and the Manning resistance factor, n. The average slope of the longest flow path is selected as the representative slope, and the average n value on the longest flow path is selected as the representative n value:

$$V \propto \frac{R^{2/3} S^{1/2}}{\bar{n}} \tag{3-2}$$

where

R = representative hydraulic radius

S = average slope of longest flow path

$\bar{n}$ = average Manning n value on longest flow path

Because the cross-sectional area of a conveyance element generally varies with the square of its hydraulic radius at capacity, the previous relationship can also be written in terms of a representative discharge, Q:

$$Q \propto \frac{R^{8/3} S^{1/2}}{\bar{n}} \tag{3-3}$$

These two relationships can be combined to eliminate the hydraulic radius and obtain:

$$V \propto \frac{Q^{1/4} S^{3/8}}{\bar{n}^{3/4}} \tag{3-4}$$

This relationship for V can be inserted into (3-1) to obtain:

$$T_L \propto \frac{L_c\ \bar{n}^{3/4}}{Q^{1/4}\ \bar{S}^{3/8}} \tag{3-5}$$

The bank-full or capacity discharge at the watershed outlet, Q_o, is selected as the representative discharge. The rational formula relates this discharge to the drainage area, an appropriate rainfall intensity, and a coefficient that accounts for other relevant factors such as land use, soils and climate:

$$Q_o = C\ i\ A \tag{3-6}$$

where

C = rational runoff coefficient

i = rainfall intensity for appropriate duration and same recurrence interval as Q_o

A = watershed area

The rainfall intensity is usually averaged over a duration equal to the watershed's time of concentration, which can be approximated as five-thirds of the lag time (NRCS 2010). In urban hydrology the time of concentration is generally 60 minutes or less. The annual exceedance probability (AEP) for bank-full or capacity flow is typically 50% or greater for natural channels and between 4% and 20% for engineered channels and enclosed conduits.

Figure 3-1 examines the relationship between rainfall intensity and duration for durations from 5 to 60 minutes and AEPs from 4% to 50% for downtown Kansas City. The data plotted in this figure were obtained from NOAA Atlas 14 (Perica et al., 2013). Figure 3-2 examines how the 50%-chance rainfall intensity varies with duration for four U.S. cities with different hydroclimates. The dashed lines in these figures are equations of the form $i = a \cdot D^x$ (in which D is duration and a and x are numerical constants) fitted to the Atlas 14 data. The fitted power-form equations approximate these i-D relationships reasonably well. These figures show the value of the exponent x for each fitted equation. Rounded to one decimal place, these exponents are all -0.5. Therefore, as a reasonable first approximation, we consider rainfall intensity to vary with the inverse square root of duration over the range of interest. Setting the rainfall duration equal to the time of concentration and approximating time of concentration as five-thirds of lag time leads to:

$$i \propto T_L{}^{-1/2} \tag{3-7}$$

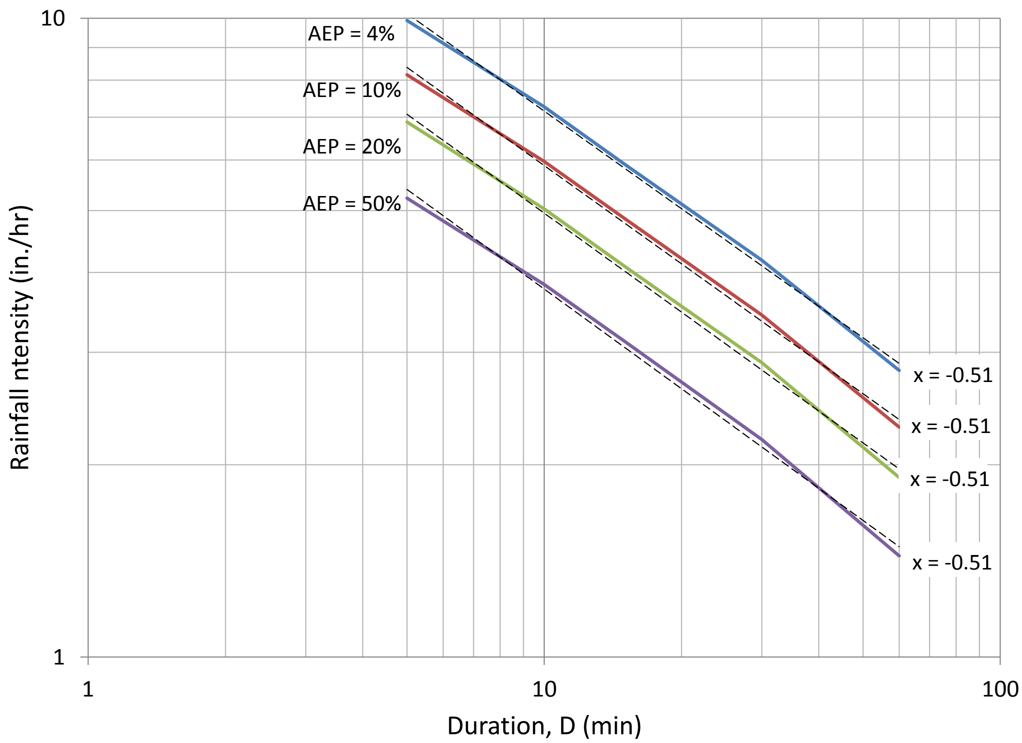

Figure 3-1. Rainfall intensity-duration relationship for Kansas City

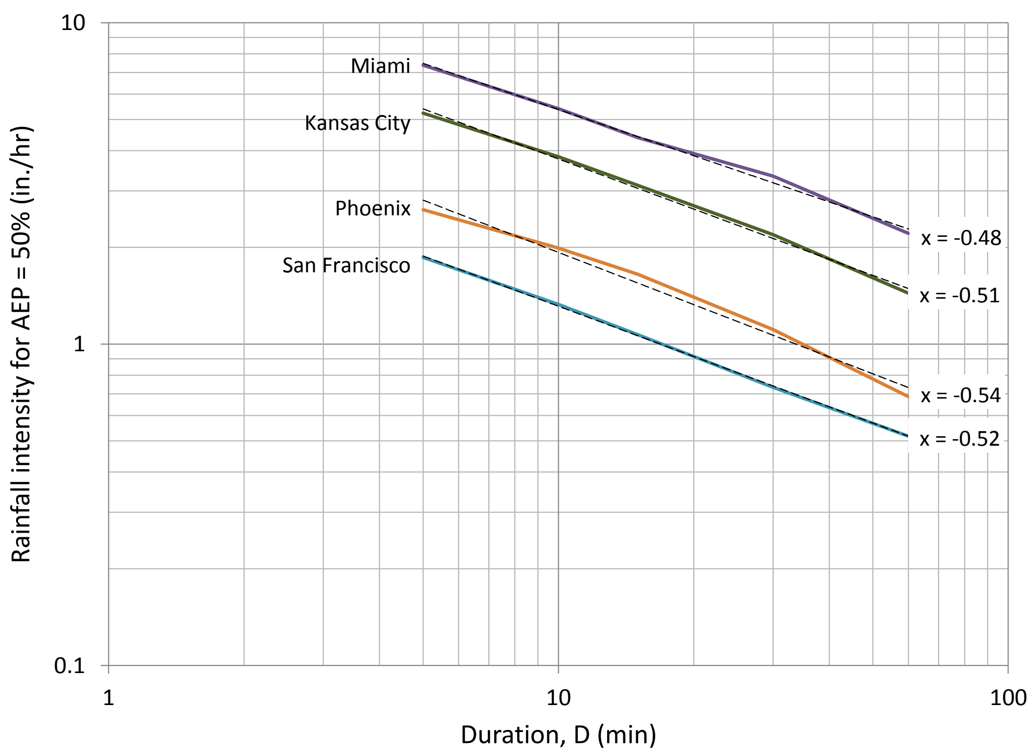

Figure 3-2. 50%-chance rainfall intensity-duration relationships for four cities

The rational runoff coefficient, C, depends on climate, soil characteristics, vegetation and land use. In urban watersheds, C is strongly dependent on the fraction of the surface area that is impervious, and the composite C for the watershed can be considered the area-weighted average of separate C values for the pervious and impervious parts of the watershed; i.e., C can be considered to vary linearly with imperviousness:

$$C \propto (1 + \beta_i \cdot R_i) \tag{3-8}$$

where

R_i = fraction of watershed area that is impervious

β_i = a dimensionless coefficient, dependent on climate and soil conditions

The constant β_i should be assigned a reasonable value based on local practice. It is helpful to note that β_i can be expressed as the quantity ($C_i/C_p - 1$) where C_p and C_i are the rational runoff coefficients for the pervious and impervious portions of the watershed, and therefore the value of β_i can be estimated from locally accepted values of C_p and C_i. In KC-APWA's Section 5600 design guidance for the Kansas City area, $C_p = 0.30$ and $C_i = 0.90$, which leads to $\beta_i = 2$.

The relationships for i and C in (3-7) and (3-8) can be inserted into the rational formula to obtain the following proportionality for Q_o:

$$Q_o \propto (1 + \beta_i \cdot R_i)\, A\, T_L^{-1/2} \tag{3-9}$$

Substituting (3-9) for Q in (3-5) and solving for T_L yields:

$$T_L \propto \frac{L^{8/7}\,\bar{n}^{6/7}}{(1+\beta_i \cdot R_i)^{2/7}\,A^{2/7}S^{3/7}} \tag{3-10}$$

The average Manning n value can be considered a length-weighted average of typical Manning n values for the natural and paved/enclosed segments of the main channel:

$$\bar{n} = n_p\,R_c + n_n\,(1 - R_c) \tag{3-11}$$

in which R_c is the fraction of the main-channel length that is paved or enclosed and n_n and n_p are representative n values for natural and paved/enclosed conditions, respectively. This relationship can be expressed as a proportionality:

$$\bar{n} \propto (1 - \beta_c\,R_c) \tag{3-12}$$

in which

$$\beta_c = 1 - \frac{n_p}{n_n} \tag{3-13}$$

Where the natural channel conditions are fairly rough and irregular, reasonable estimates for the n_p/n_n and β_c would be 1/4 and 3/4.

Substituting (3-12) for $\bar{n}$ in (3-10) yields:

$$T_L \propto \frac{L^{8/7}\,(1 - \beta_c\,R_c)^{6/7}}{(1+\beta_i \cdot R_i)^{2/7}\,A^{2/7}S^{3/7}} \tag{3-14}$$

This relationship can be simplified by defining the quantity A/L as the average width of the watershed, W, and substituting W·L for A to obtain:

$$T_L \propto \left[\frac{L}{\sqrt{S}} \cdot \frac{(1 - \beta_c\, R_c)}{(1 + \beta_i \cdot R_i)^{1/3}\, W^{1/3}}\right]^{6/7} \tag{3-15}$$

This proportionality can be rewritten as an equation by inserting a constant "k" in front of the bracketed term. The value of this constant might depend to some extent on local conditions and therefore should be calibrated with local data if possible.

$$T_L = k\left[\frac{L}{\sqrt{S}} \cdot \frac{(1 - \beta_c\, R_c)}{(1 + \beta_i \cdot R_i)^{1/3}\, W^{1/3}}\right]^{6/7} \tag{3-16}$$

For rural watersheds with natural channel conditions and negligible imperviousness, the constants R_i and R_c can be set to zero, which leads to a simpler relationship:

$$T_L = k\left(\frac{L}{\sqrt{S}} \cdot \frac{1}{W^{1/3}}\right)^{6/7} \tag{3-17}$$

3.2. Calibration of the general semi-analytical formula for the Kansas City area

Equation (3-16) was calibrated to the dataset of Table 2-3 by the least-squares method. In this calibration, β_i was set to 2 and β_c was set to 3/4. Both sides of the equation were transformed to a natural-log scale to make the variance in error more uniform. The log transformed equation is:

$$\ln(T_L) = \frac{6}{7}\ln(x) + \ln(k) \tag{3-18}$$

in which x is the term inside the brackets in (3-16). We applied this equation to the 30 watersheds and found the value of k that minimized the sum of the squares of the errors in

$\ln(T_L)$. The best-fit value of k was found to be 0.0163. After transforming back to a normal scale we get

$$T_L = 0.0163 \left[\frac{L}{\sqrt{S}} \cdot \frac{(1 - 0.75R_c)}{(1 + 2R_i)^{1/3}\,W^{1/3}}\right]^{6/7} \qquad (3\text{-}19)$$

Equation (3-19) has a coefficient of determination (R^2) of 0.910 and a standard error of 0.269 in natural-log units or (+30.8%, -23.6%).

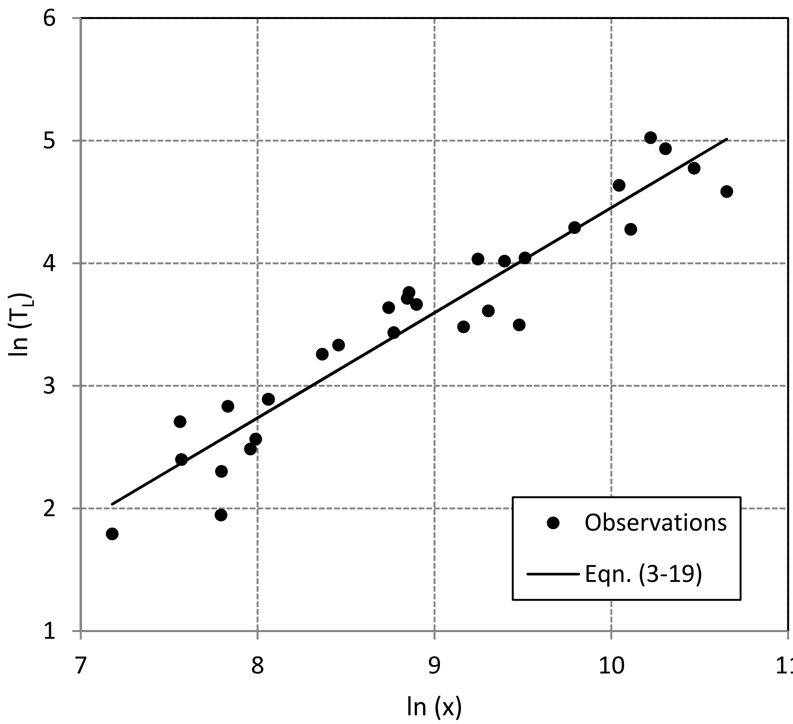

Figure 3-3. Observed data and best fit Equation (3-19) in natural-log scale

Chapter 4

Regression Analysis

4.1. Variable selection and correlations

The regression model chosen for the estimation of lag time was a power function of form

$$T_L = a \cdot x_1^{b_1} \cdot x_2^{b_2} \cdot x_3^{b_3} \cdot \ldots \cdot x_n^{b_n} \tag{4-1}$$

where x_1, x_2, x_3, … are independent variables; b_1, b_2, b_3, … are regression coefficients; and a is the regression constant. The model in (4-1) was fitted to the data of Table 2-3 by performing least-square linear regression on the natural logarithms of the selected variables. All of the watershed characteristics in Table 2-1 except R_c and R_i were considered as predictor variables for the regression. If R_c and R_i were included, the exponents on these terms would be negative and the equation would give an infinite lag time for $R_c = 0$ or $R_i = 0$. Instead, the terms $(1 – 0.75R_c)$ and $(1 + 2R_i)$ were used to account for the effects of land development and channel modification. These terms appear in the semi-analytical equation developed in Chapter 3.

The statistical program StatistiXL, which runs within Excel, was used to perform a correlation analysis on the transformed variables to examine which ones could be more influential or cause problems in the regression. A high correlation coefficient (either positive or negative) with lag time indicates a strong linear relationship, while high correlation between predictor variables can produce larger standard errors and uncertainty in the regression analysis. The correlation matrix is shown in Table 4-1.

Table 4-1. Correlation matrix for the independent and dependent variables

	ln(L)	ln(L_c)	ln(L_{avg})	ln(A)	ln(W)	ln(S)	ln(1–0.75R_c)	ln(1+2R_i)	ln(T_L)
ln(L)	1.00								
ln(L_c)	0.97	1.00							
ln(L_{avg})	0.99	0.97	1.00						
ln(A)	0.94	0.87	0.94	1.00					
ln(W)	0.70	0.58	0.71	0.90	1.00				
ln(S)	-0.82	-0.76	-0.82	-0.79	-0.61	1.00			
ln(1–$\beta_c R_c$)	0.69	0.64	0.71	0.62	0.43	-0.71	1.00		
ln(1+$\beta_i R_i$)	-0.57	-0.55	-0.57	-0.53	-0.39	0.67	-0.58	1.00	
ln(T_L)	0.87	0.84	0.87	0.79	0.56	-0.87	0.89	-0.66	1.00

*High correlation coefficients with lag time are highlighted.

Our analysis shows that all the measures of length are highly correlated with lag time. To a precision of two decimal places, L and L_{avg} have the same correlation coefficient, and L_c has a slightly lower value. We chose to use L rather than L_{avg} in the regression because its value is easier to determine. The other two variables that are highly correlated with lag time are (1 – 0.75R_c) and S. It is noteworthy that the variables (1 – 0.75R_c) and (1 + 2R_i) are not highly correlated to each other or with L or S.

4.2. Regression analysis

A principal component analysis (using StatistiXL) was performed on the variables L, S, W, (1 – 0.75R_c) and (1 + 2R_i) in an attempt to reduce the correlations among independent variables. All of the five components were extracted. Each principal component is a linear combination of the standardized variables multiplied by their respective score coefficients. The score coefficients of the principal components and how much of the variance they explain can be seen in Table 4-2.

Table 4-2. Principal component score coefficients and explained variance

Variable	PC 1	PC 2	PC 3	PC 4	PC 5
ln(L)	0.488	0.212	0.173	0.393	0.729
ln(W)	0.399	0.734	-0.253	-0.457	-0.174
ln(S)	-0.492	0.047	-0.008	-0.592	0.637
$\ln(1–0.75R_c)$	0.439	-0.338	0.664	-0.494	-0.086
$\ln(1+2R_i)$	-0.409	0.548	0.682	0.206	-0.158
Eigenvalue	3.49	0.67	0.43	0.25	0.16
% of variance explained	69.86	13.43	8.54	4.92	3.25
Cumulative % of variance explained	69.86	83.29	91.83	96.75	100.00

A forward stepwise linear regression was performed using the principal components as predictor variables. After transforming back to the initial variables, the resulting equation was

$$T_L = 0.201 L^{0.374} W^{-0.021} S^{-0.480} (1 - 0.75R_c)^{1.724} (1 + 2R_i)^{-0.203} \qquad (4\text{-}2)$$

This equation included the first three principal components, which were all found to be significant. It has a coefficient of determination (R^2) of 0.932, an adjusted R^2 of 0.924 and a standard error of 0.242 in natural-log units. The variable W has a very small exponent, making it insignificant in the transformed equation. On the other hand, the term $(1 – 0.75R_c)$ has the largest exponent, showing its importance for the estimation of lag time. Adding the fourth principal component to the regression generates a similar equation with identical statistics, with the exception that $(1 + 2R_i)$ becomes the insignificant variable. A problem with (4-2) is that it lacks some physical logi7c. Specifically, the impact of channel paving is exaggerated. T_L for a fully paved channel ($R_c = 1$) is only 9% of the T_L for a completely natural channel ($R_c = 0$).

Additionally, a forward stepwise linear regression was performed on the same five variables without transforming them into principal components. The resulting equation was

$$T_L = 0.0675L^{0.431}S^{-0.531}(1 - 0.75R_c)^{1.577} \tag{4-3}$$

which includes variables with P-values of 0.006 or less. Low P-values correspond with statistical significance. Equation (4-3) has an R^2 of 0.933, an adjusted R^2 of 0.925 and a standard error of 0.241 natural log units. The fact that Equation (4-3) has almost the same statistics as (4-2) indicates that most of the information needed to estimate lag time is contained in the variables L, S, and $(1 – 0.75R_c)$. However, (4-3) has the same problem as (4-2). T_L for $R_c = 1$ is only 11% of T_L for $R_c = 0$.

If we consider Manning's equation for uniform flow, velocity is directly proportional to the square root of slope and inversely proportional to Manning's n coefficient (a measure of channel roughness), which in turn means that travel time (T_t) is directly proportional to Manning's n and length and inversely proportional to the square root of slope.

$$T_t \propto a\left(\frac{L\bar{n}}{\sqrt{S}}\right)^b \tag{4-4}$$

This logic suggests a lag-time equation of the following form:

$$T_L = a\left[\frac{L}{\sqrt{S}} \cdot (1 - 0.75R_c)\right]^b \qquad (4\text{-}5)$$

Equation (4-5) was fitted to the data by least-squares linear regression performed on the natural-log transformations of lag time and the term inside the brackets. The resulting equation was

$$T_L = 0.0029\left[\frac{L}{\sqrt{S}} \cdot (1 - 0.75R_c)\right]^{0.80} \qquad (4\text{-}6)$$

Equation (4-6) has a R^2 of 0.896 and a standard error of 0.289 log units or (+33.5%, -25.1%). Other values of β_c were tested as a way of calibrating the equations further. Higher values of β_c resulted in equations with slightly better measures of fit. However, higher values of β_c correspond to unrealistically low values of the ratio n_p/n_n. A value of $\beta_c = 0.75$ ($n_p/n_n = 1/4$) was judged to be reasonable and produced an equation with an acceptable fit.

Chapter 5

Recommended Equations for Lag Time and Time of Concentration

5.1. Recommended equations for use in the Kansas City Metro area

The semi-analytical equation (3-19) is preferable to Equation (4-6). Equation (3-19) fits the data better than (4-6), and more importantly it accounts for the watershed's imperviousness and width. Physical reasoning suggests that these variables are relevant because they directly affect the discharge in the channel, and the higher the discharge, the shorter the lag time. In addition, (3-19), has a stronger theoretical basis than a regression of the power form. For these reasons, we recommend using the semi-analytical equation (3-19) to estimate lag time in the Kansas City metropolitan area.

The recommended equation for time of concentration follows from (3-19) and the NRCS-recommended approximation $T_L = 0.6\ T_C$:

$$T_C = 0.0271 \left[\frac{L}{\sqrt{S}} \cdot \frac{(1 - 0.75\ R_c)}{(1 + 2\ R_i)^{1/3}\ W^{1/3}} \right]^{6/7} \tag{5-1}$$

5.2. Comparison of new lag time equation to previous equation for the Kansas City area

KDOT's current lag-time equation for urban watersheds, developed from an earlier analysis of Johnson County ALERT-system data (McEnroe and Zhao, 2001), is:

$$T_L = 0.0087\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74} e^{-3.5R_i} \qquad \text{for } 0 \leq R_i \leq 0.4 \tag{5-2}$$

where T_L is the lag time in minutes, L is the length of the longest flow path in feet, $S_{10\text{-}85}$ is the average slope between the 10% and 85% points on the longest flow path in feet per foot, and R_i is the impervious area ratio. Equation (5-2) was developed from data for 14 sites in Johnson County (three of which were used in this study). Equation (5-2) was applied to the dataset of Table 2-3 along with (3-19) and (4-6), and the three computed estimates of T_L were plotted against the observed lag times in Figure 5-1 and Figure 5-2. From visual examination, Equation (5-2) tends to deviate more from the observed lag times and slightly underestimate them. This underestimation appears more pronounced for the watersheds with shorter lag times (T_L < 1 hour). This is probably because fewer watersheds with short lag times were used on the development of (5-2). The average lag time for watersheds used to generate (5-2) was 81 minutes, while the average lag time for the watersheds in this study is 47 minutes.

5.3. Limitations of new equations

Equations (3-19) and (5-1) are applicable to watersheds with characteristics that do not vary greatly from those of the watersheds in our data set. Table 5-1 shows the ranges of values for the five inputs to the new equations.

Table 5-1. Ranges of variables in data set

Variable	Range
L	0.9 – 11 miles
S	0.4% – 2%
W	0.2 – 1.4 miles
R_c	0 – 0.75
R_i	0.01 – 0.50

Because the equations have a solid physical basis, they should give reasonable results for smaller watersheds and for more densely developed watersheds. These equations should be applied only to watersheds that do not contain impoundments of a size that would alter the flood hydrograph to a significant degree. Impoundments increase the response time of a watershed; (3-19) would tend to underestimate the lag times of watersheds with significant impoundments. Equations (3-19) and (5-1) do not account for the spatial distribution of development within the watershed. A watershed with development concentrated at the lower end would have a shorter lag time than the same watershed with development concentrated at the upper end. Reasonable judgement should be used when applying these equations to such watersheds. In areas where soil and natural channel conditions are much different from than the ones in the Kansas City metropolitan area, the values 2 and 0.75 for β_i and β_c, respectively, might not be accurate and should be adjusted prudently.

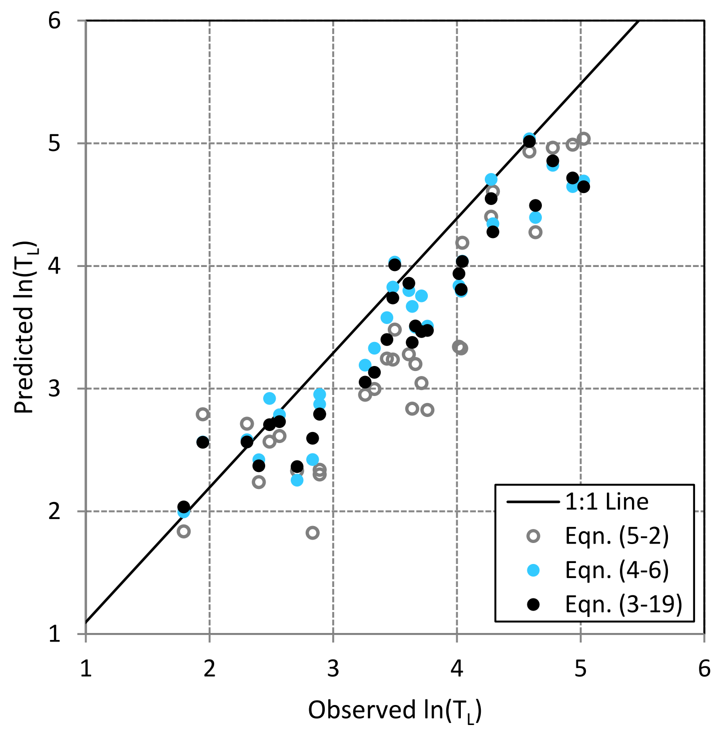

Figure 5-1. Comparison of lag time equations (natural log scale)

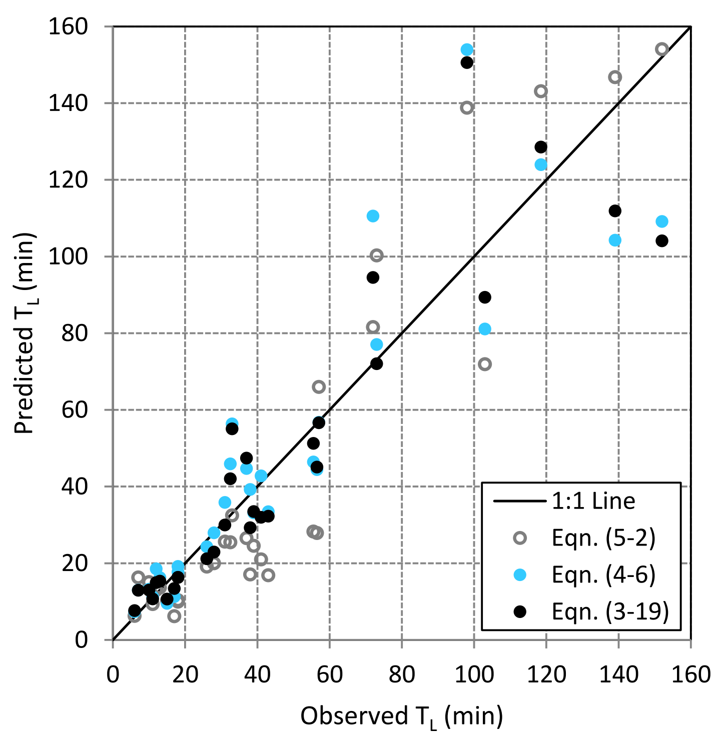

Figure 5-2. Comparison of lag time equations (normal scale)

Table 5-2. Comparison of calculated and observed lag times

	Lag times (minutes)			
Site ID	Observed	Eqn. (3-19)	Eqn. (4-6)	Eqn. (5-2)
1140	32.5	42.1	45.9	25.5
1400	41	32.0	42.8	21.0
1450	18	16.3	17.7	10.4
1650	26	21.2	24.3	19.1
1680	6	7.7	7.4	6.3
2090	73	72.0	77.0	100
2220	152	104	109	154
2540	57	56.6	56.8	66.0
2600	15	10.6	9.5	10.3
2640	11	10.7	11.3	9.4
2700	18	16.3	19.2	10.0
2720	7	13.0	13.0	16.3
2730	10	13.0	13.2	15.1
3020	13	15.3	16.2	13.6
3160	38	29.3	39.2	17.1
3170	98	151	154	139
3250	72	94.6	111	81.6
3310	118.5	129	124	143
3350	139	112	104	147
3660	103	89.4	81.1	71.9
3690	39	33.5	33.1	24.6
3720	43	32.3	33.4	16.9
3840	55.5	51.3	46.4	28.3
3900	37	47.4	44.7	26.6
3940	33	55.1	56.4	32.5
3980	56.5	45.1	44.4	27.9
4080	31	30.0	35.8	25.6
4150	17	13.4	11.3	6.2
5050	28	22.9	27.9	20.0
5700	12	15.0	18.5	13.0

Chapter 6

Conclusions

From the work presented in this report we draw the following conclusions:

1. Roughness of the main channel is very influential on lag time. The percentage of the main-channel length that is enclosed or paved is more important than the imperviousness of the watershed in determining the lag time.
2. Ratios of measures of channel modification and watershed development, R_c and R_i respectively, should not be included directly as independent variables in regression analysis with a power function as model. If either variable is 0, then the equation would result in a physically impossible lag time. Before including these two important variables in our regression model, we applied transformations suggested by our semi-analytical model.
3. The general semi-analytical formula for urban watersheds developed in this report is recommended for the estimation of T_L and T_C. It was selected over other methods for its inclusion of relevant factors and theoretical basis. It is derived from the Manning equation for hydraulic friction, the rational equation for peak flow, and a rainfall intensity-duration relationship. It accounts for the length, slope and roughness of the main channel and the average width and imperviousness of the watershed.
4. The new recommended equations for T_L and T_C have some advantages over the methods currently specified in KC-APWA's Section 5600 design guidance and the KDOT Design

Manual. The new equations are simpler to apply than the velocity method and the values of the five inputs are easier to determine. Unlike the current equations for urban lag time and time of concentration in the KDOT Design Manual, the new equations account for very significant effects of enclosed conduits and lined channels.

5. Our approximate analysis of the 30 watersheds in our dataset, based on the assumption that travel time is directly proportional to flow path distance over the average flow velocity in the watershed, lends some support to the NRCS-recommended relationship $T_L = 5/3\ T_C$. Our recommended equation for time of concentration makes use of this widely accepted approximation. However, we note that the value of T_L/T_C for a given watershed depends on the watershed's shape, the spatial distribution of urban development, and other factors. This is a matter that requires further study.

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Additional sources for Figure 2-1:

- U.S. Geological Survey, 201403, USGS Small-scale Dataset - 1:1,000,000-Scale Major Roads of the United States 201403 FileGDB 10.1: U.S. Geological Survey, Rolla, MO.

- U.S. Geological Survey, National Geospatial Technical Operations Center, 20140401, USGS National Boundary Dataset (NBD) for Missouri 20140401 State or Territory FileGDB 10.1: U.S. Geological Survey, Reston, VA.

Appendix

Calibration Results for Gaged Watersheds

Gage 1140				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	03/09/09	23	1.06	7.61
2	05/15/09	29	1.50	6.89
3	06/16/10	39	0.95	8.43
4	07/11/10[A]	30	0.79	8.39
5	09/23/10	45	0.83	5.88
6	05/21/11	43	0.67	7.50
7	06/19/11	31	1.38	9.49
8	08/19/11	28	1.65	7.62
9	05/31/13	34	2.80	13.01
10	10/02/14	38	0.83	4.60
Median		32.5		

[A] First event of the day.

Gage 1400				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/05/01	43	2.50	14.82
2	08/25/01	41	1.34	10.62
3	05/24/02	34	2.21	9.83
4	06/22/03	39	2.37	12.92
5	07/11/06	65	3.43	11.49
6	07/09/07	41	2.13	13.67
7	08/08/07	42	2.72	12.49
8	04/27/14	48	1.08	10.18
9	07/07/14	40	1.02	5.33
10	09/17/14	34	1.18	4.96
Median		41		

Gage 1450				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/05/08	22	1.42	6.80
2	06/10/09	18	1.62	7.76
3	07/12/09	19	0.71	5.98
4	07/11/10	18	1.05	8.92
5	07/20/10	18	1.38	7.39
6	05/31/13	31	1.73	6.96
7	07/07/14	15	1.06	3.87
8	08/07/14	15	1.38	3.80
Median		18		

Gage 1650				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/27/06	19	4.41	11.34
2	10/15/08	30	1.10	12.02
3	06/19/11	19	1.38	7.89
4	08/22/11	23	2.17	9.97
5	05/06/12	16	2.88	10.25
6	05/31/13	15	2.92	12.05
7	04/27/14	29	1.24	8.47
8	07/08/14	33	0.99	5.33
9	09/17/14	30	0.95	6.18
10	10/02/14	31	0.83	5.66
Median		26		

Gage 1680				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	10/04/98	6	0.71	3.47
2	06/05/01	6	1.69	4.49
3	08/26/05	5	1.35	3.45
4	07/11/06	9	4.1	5.93
5	08/27/06	6	3.94	4.37
6	06/03/08	6	1.26	3.12
7	07/29/08	6	6.70	4.77
8	06/14/10	6	2.40	4.15
9	05/31/13	10	2.68	5.9
10	06/15/13	4	2.64	4.86
Median		6		

Gage 2090				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/04/01	70	1.18	6.97
2	05/24/02	80	1.66	8.22
3	06/10/04	84	1.00	7.00
4	06/04/05	82	3.75	8.83
5	08/27/06	76	4.14	8.18
6	06/04/08	70	3.47	10.41
7	06/12/10	65	2.56	9.79
8	07/20/10	69	1.42	8.30
9	06/19/11	61	1.62	7.66
10	05/31/13	96	1.58	8.90
Median		73		

Gage 2220				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/26/99	150	1.32	9.48
2	06/04/01	195	1.42	10.88
3	03/04/04	125	4.62	10.47
4	05/19/04	152	3.00	10.81
5	08/24/04	232	2.49	10.32
6	06/04/05	173	2.92	10.27
7	06/03/08	104	3.83	12.05
Median		152		

Gage 2540				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	03/10/10	81	0.75	7.83
2	04/06/10	39	1.77	14.56
3	04/09/10	55	2.32	10.95
4	04/22/11	82	1.26	8.59
5	05/28/11	54	0.98	10.26
6	05/30/13	57	1.29	9.87
7	04/27/14[A]	85	0.51	10.13
Median		57		

[A] First event of the day.

Gage 2600				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/21/11	18	1.62	2.92
2	05/24/11[A]	14	0.91	1.85
3	05/24/11[B]	14	0.75	1.66
4	05/25/11	16	1.10	2.42
5	08/18/11	16	1.08	2.12
6	02/28/12	9	1.38	2.76
7	06/15/13	13	1.57	2.58
8	02/19/14[B]	15	0.81	2.13
9	04/27/14	17	1.05	2.52
Median		15		

[A] First event of the day. [B] Second event of the day

Gage 2640				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/06/11	12	1.42	3.04
2	08/18/11	10	1.57	3.28
3	09/19/13	11	2.11	3.42
Median		11		

Gage 2700				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/16/11	22	1.42	0.92
2	05/06/12[A]	14	1.20	0.79
3	05/31/13	18	1.85	1.10
Median		18		

[A] First event of the day.

Gage 2720				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/19/11	6	1.16	2.40
2	05/27/13	14	2.36	1.19
3	05/31/13	7	1.41	1.52
Median		7		

Gage 2730				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	05/25/11	9	0.98	2.58
2	08/20/11	11	1.50	2.97
3	05/06/12	10	1.15	2.16
4	04/08/13	13	0.61	2.40
5	05/31/13	14	1.20	3.27
6	07/03/13	7	1.20	2.02
7	09/18/13	9	2.09	2.01
Median		10		

Gage 3020				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/07/00	13	1.15	5.74
2	08/13/05	7	1.67	7.65
3	08/08/07	21	3.02	6.57
4	06/03/08	13	1.32	6.33
5	06/24/09	14	1.53	5.70
6	04/04/10	18	1.03	5.93
7	08/06/14	13	1.14	2.26
Median		13		

Gage 3160				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/09/07	48	2.96	8.78
2	07/29/08	30	2.57	7.80
3	09/12/08	33	2.35	8.84
4	06/10/09	39	1.62	8.20
5	06/13/10	44	3.39	9.94
6	07/28/10	38	3.68	7.83
7	07/07/14	34	1.18	3.39
8	08/06/14[A]	35	1.50	3.52
9	08/06/14[B]	39	0.95	3.68
Median		38		

[A] First event of the day. [B] Second event of the day.

Gage 3170				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/06/04	111	2.87	15.68
2	08/20/05	98	2.64	14.22
3	08/09/07	59	4.49	14.83
4	04/06/10	83	1.34	14.13
5	06/14/10	99	2.64	15.65
Median		98		

Gage 3250				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/20/05	72	3.25	11.60
2	06/11/09	63	1.67	11.06
3	06/15/09	72	1.20	11.35
4	06/14/10	63	2.46	13.19
5	06/19/11	68	1.14	10.96
6	08/22/11	88	1.22	10.89
7	05/06/12	115	1.30	11.10
8	05/31/13	72	1.72	11.64
Median		72		

Gage 3310				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/25/05	105	1.84	9.82
2	05/06/07	107	1.54	9.35
3	06/02/08	130	1.85	9.99
4	06/03/08	132	1.49	11.95
5	06/14/10	107	1.81	10.65
6	09/02/14	170	1.69	7.17
Median		118.5		

Gage 3350				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/20/05	139	3.23	9.23
2	08/26/05	109	1.38	9.74
3	05/06/07	123	1.93	9.22
4	06/02/08	155	2.48	10.88
5	06/04/08	162	1.46	11.33
6	04/06/10	139	1.34	9.79
7	05/25/11	121	1.26	9.14
8	05/31/13	163	2.09	11.18
Median		139		

Gage 3660				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/21/10[A]	104	0.55	6.01
2	04/22/11	103	1.18	6.94
3	05/25/11[A]	98	1.14	3.73
4	05/25/11[B]	82	0.75	4.78
5	08/19/11	87	1.26	5.44
6	08/20/11	99	1.30	6.04
7	02/28/12	115	0.87	4.57
8	05/27/13	110	1.81	6.27
9	05/30/13	144	0.98	5.63
10	06/28/13	116	0.91	4.85
11	09/19/13	94	2.13	5.50
12	10/04/13	90	1.30	5.76
13	06/12/14	154	0.87	5.56
Median		103		

[A] First event of the day. [B] Second event of the day.

Gage 3690				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/22/11	39	1.38	3.40
2	05/24/11[A]	43	0.83	2.07
3	06/26/11	43	1.65	2.23
4	12/19/11	42	1.22	2.09
5	02/28/12	32	1.50	3.58
6	06/15/13	37	1.34	2.87
7	04/27/14	33	1.30	3.18
Median		39		

[A] First event of the day.

Gage 3720				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/18/10	41	0.79	1.11
2	09/22/10[A]	57	0.87	0.79
3	09/23/10	43	0.83	1.07
Median		43		

Gage 3840				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/12/11	56	1.30	1.53
2	08/19/11	49	0.79	2.03
3	05/31/13	58	1.50	1.44
4	06/15/13	57	1.18	1.09
5	09/19/13	53	2.09	1.00
6	04/24/14	64	0.83	1.13
7	04/27/14	54	1.58	1.64
8	06/02/14	55	1.02	1.27
Median		55.5		

Gage 3900				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/01/10	37	2.40	3.81
2	07/12/11	40	1.65	4.07
3	08/19/11	35	1.69	4.34
4	02/28/12	41	1.30	3.98
5	05/31/13	35	1.89	5.23
6	06/15/13	44	1.50	4.11
7	04/24/14	45	1.38	3.86
8	04/27/14	33	1.58	3.87
9	06/01/14	37	1.30	3.51
Median		37		

Gage 3940				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/18/10	15	3.82	6.41
2	09/21/10	36	0.98	3.05
3	07/07/11	32	1.26	3.34
4	08/18/11	33	0.63	3.37
5	05/30/13	61	1.97	5.37
6	05/31/13	32	1.81	7.05
7	06/15/13	38	1.38	5.20
8	04/24/14	67	1.02	4.11
9	06/02/14	30	1.22	3.21
Median		33		

Gage 3980				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/22/11	61	1.22	0.75
2	09/19/13	52	3.23	1.43
Median		56.5		

Gage 4080				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	05/31/13	32	0.98	3.00
2	09/19/13	18	2.60	2.19
3	05/27/14	27	2.17	1.95
4	06/02/14	31	0.75	1.30
5	06/12/14	32	1.10	1.22
Median		31		

Gage 4150				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/14/11	23	1.14	1.14
2	04/22/11	18	1.34	2.91
3	05/20/11	21	0.63	1.02
4	05/24/11	17	0.67	1.67
5	05/25/11	17	0.83	1.97
6	08/18/11	17	1.58	1.00
7	06/27/13	12	1.58	1.00
Median		17		

Gage 5050				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/04/08[A]	20	1.46	7.70
2	06/04/08[B]	28	1.73	8.45
3	06/05/08	35	1.10	7.39
4	06/09/09	28	2.21	7.83
5	06/16/10	34	0.87	7.00
6	08/20/10	18	2.25	7.56
7	08/20/11	23	1.50	7.44
8	05/06/12	29	2.21	7.54
9	05/31/13	28	1.50	8.97
10	07/07/14	21	1.22	6.94
11	08/06/14	23	3.15	9.45
12	10/01/14	34	0.61	5.93
Median		28		

[A] First event of the day. [B] Second event of the day.

5700				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/13/05[C]	12	2.13	7.30
2	04/25/07	7	1.58	7.08
3	09/07/07	13	1.34	9.64
4	09/25/07	9	0.83	9.52
5	09/30/07	15	0.55	6.70
Median		12		

[C] Third event of the day.